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**DIRECT EMBEDMENT ANCHOR HOLDING
CAPACITY**

R. J. Taylor, et al

Naval Civil Engineering Laboratory

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By

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Port Hueneme, CA 93043

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DIRECT EMBEDMENT ANCHOR HOLDING CAPACITY

Technical Note N-1245

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R. J. Taylor and H. J. Lee

ABSTRACT

Techniques for predicting the maximum uplift forces which may be applied to direct embedment anchors without causing the anchor to pull out are provided. This holding capacity problem is subdivided into three categories: immediate breakout, long-term static load, and long-term repeated load. Holding capacities under long-term repeated and long-term static loading conditions are poorly understood at present. It was therefore necessary to combine work from other areas with a small amount of directly applicable work to yield approximate immediate use results. For each manner of loading considered, two general types of seafloors are considered: cohesionless and cohesive soil. Rock is not considered in this report.

To simplify the holding capacity prediction process, the suggested procedure is outlined without rationale in a block diagram with each item of the diagram being briefly discussed. A sample problem is also presented.

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	LINE A		LINE B		LINE C	
	ROLE	DT	ROLE	DT	ROLE	DT
Direct Embedment Anchors						
Anchor Holding Capacity						
Breakout Forces						
Loads 'Forces'						
Cohesive Soils						
Noncohesive Soils						
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Long-Term Loading						

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INTRODUCTION

The purpose of this report is to provide techniques for predicting the maximum uplift forces which may be applied to direct embedment anchors without causing the anchors to pull out. These forces will be identified as the anchor holding capacities. Since holding capacity is not a property of a particular anchor but may vary considerably with seafloor type, embedment depth, and method of loading, this report shows how these factors influence holding capacity and how to design anchors conservatively in a variety of situations.

It is necessary to subdivide the holding capacity problem into categories. The first subdivision is based on method of loading of which three will be considered:

- (a) Immediate breakout
- (b) long-term static load
- (c) long-term repeated load

Immediate breakout describes the situation in which the anchor is loaded as rapidly as possible until breakout occurs. Most field tests have been conducted in this manner, and most of the theoretical results are directed toward it. This loading method is presented first because long-term holding capacities are usually presented as fractions of the immediate capacity. Long-term static holding capacity refers to the situation in which an anchor pulls out after a constant upward force has been applied over a long period of time. This holding capacity would be associated with moored objects such as submerged buoys. Repeated loading involves a line force which varies considerably with time and which can be approximated by a sinusoidally varying force with a certain period and amplitude. Moored surface buoys and ships can provide this type of force application. For each manner of loading two general types of seafloors are considered: cohesionless and cohesive soil. Rock is not considered in this report.

IMMEDIATE HOLDING CAPACITY

The commonly used equation for representing the holding capacities of embedment anchors is the following (Vesic, 1969):

$$F_T = A (c\bar{N}_c + \gamma_b D\bar{N}_q) \quad (1)$$

where F_T = holding capacity (lbs)

A = fluke area (ft)²

c = cohesion of soil (psf)

γ_b = buoyant unit weight of soil (pcf)

D = fluke embedment depth (ft)

\bar{N}_c, \bar{N}_q = holding capacity factors

The equation is relatively general and can be applied approximately to almost any form of loading. However, the holding capacity factors and the cohesion may vary with loading mode and have been found to vary with soil type, density, and relative anchor embedment depth, D/B , where B is the fluke width. The major problem of estimating holding capacity is then one of estimating c , \bar{N}_c , and \bar{N}_q .

Before discussing methods for estimating these factors, it should be noted that Equation 1 refers to square or circular flukes. In order to account for rectangular flukes, the following relation derived from bearing capacity equations (Skempton, 1951, Hansen, 1957, and Meyerhoff, 1951) is suggested:

$$F_T = A (c\bar{N}_c + \gamma_b D\bar{N}_q) (0.84 + .16 B/L) \quad (2)$$

where B = fluke diameter or width

L = fluke length

Compatible units should be used in all equations of this form.

Cohesive Soils

The strength of soils is generally given by the Mohr-Coulomb equation:

$$\tau_f = c + N \tan \phi \quad (3)$$

where τ_f = shear strength

c = cohesion

N = normal force on failure plane

ϕ = internal friction angle

The equation states simply that soils may be partly frictional and partly cohesive in their response. With cohesionless soils (sands), the behavior is strictly frictional ($c = 0$), while with cohesive soils

(clays), the behavior may be both cohesive and frictional, depending upon the time factor. If loading is slow or long-term, cohesive soils behave in a frictional manner somewhat similar to sands. However, when loading is rapid, the behavior is quite different in that the frictional element disappears (ϕ appears equal to 0) and the cohesive element becomes equal to the shear strength. For this case, the holding capacity factor \bar{N} (which is the frictional factor) reduces to 1.0 and the cohesion, c , becomes the measured short-term shear strength. In a later section, methods for estimating and averaging soil strength properties will be given. For cohesive soils, Equation 2 reduces to

$$F_T = A (c\bar{N}_c + \gamma_b D) (0.84 + .16 B/L) \quad (4)$$

The only remaining problem in estimating the short-term holding capacity in cohesive soil is that of estimating \bar{N}_c . This quantity has been found to increase almost linearly as a function of D/B , reaching a constant final value of around $\bar{N}_c = 9$ (Ali, 1969; Kupferman, 1971; and Adams and Hayes, 1967), at certain D/B values which appear to be functions of soil shear strength. The point at which \bar{N}_c becomes independent of D/B is usually identified as the point of separation between "shallow" and "deep" anchor behavior. These terms will be discussed in greater detail later.

Simplified results of short-term, small-scale pullout tests in clay are plotted in Figure 1. The test data are somewhat limited and scattered (scatter not shown to simplify diagram); however, the plot does illustrate the variation of \bar{N}_c with the average shear strength. Conservative approximations to the data are presented in Figure 2 and can be represented by the relations:

$$\bar{N}_c = 3.8 (D/B) \left(\frac{c}{c} + 0.3\right) \quad (5)$$

or $\bar{N}_c = 9$, whichever is smaller, for $0.75 \text{ psi} \leq c \leq 4 \text{ psi}$.

If c is less than 0.75 psi or greater than 4 psi, engineering judgment indicates that it should be assumed equal to 0.75 psi or 4 psi, respectively, for purposes of calculating \bar{N}_c .

The suggested technique for estimating the short-term holding capacity of an embedment anchor in cohesive soil is to use Equation 4 with \bar{N}_c obtained from Equation 5.

This procedure appears to be valid for predicting short-term holding capacities and in designing anchors. However, when the results of field anchor tests are to be evaluated, it is necessary to consider another factor which is usually identified as suction.

When a load is first applied to an anchor embedded in soil, it may be carried either by shearing stresses in the soil over the anchor or by negative gage pressures (suction) in the water contained by the soil beneath the anchor. In time the suction pressures will dissipate and thereby decrease the holding capacity. This is almost strictly a problem

associated with short-term capacities in cohesive soil since suction pressures dissipate almost immediately in sand. Also it is a problem associated primarily with field tests since laboratory tests, such as those presented in Figure 1, are usually performed with anchor bottoms vented to eliminate suction. It is important to recognize that short-term field test results may be unconservatively high, and it would be desirable to be able to predict the extent of these suction forces so that they may be subtracted from the measured holding capacities to yield more reliable results.

Only limited research has been conducted to investigate the magnitude of suction forces. Papers published at Duke University (Vesic, 1969, and Ali, 1968) and the University of Massachusetts (Kupferman, 1971) mention the suction effect as important but do not provide information on how to evaluate it.

The NCEL research on breakout of partially embedded objects is somewhat applicable because suction is thought to be the major contributor in this form of breakout. As a result of this research (NCEL, 1972) it was concluded that the significant parameters in partially embedded object breakout are the soil shear strength, c , and the relative embedment depth, D/B . For immediate breakout to occur, rupturing of the soil beneath the object is required, thus the significance of c . It was found that for a relative embedment depth of 1.0, the breakout force was equal to about $7 A c$, where A is the object plan area. It appears that this is a maximum value for the suction force and that further increases with depth will not occur.

Research of the Hydro-Electric Power Commission of Ontario (Adams and Hayes, 1967) provides additional information on the soil suction problem. Laboratory tests performed with soft ($c = 1.5$ to 2.0 psi) clay and vented and unvented flukes yielded results which compare favorably to the NCEL results. The data indicate that the suction effect at D/B ratios of approximately 3 and 4.5 is about $7 A c$, which is equivalent to stating that the holding capacity coefficient attributed to suction is about 7.

The NCEL and Ontario Hydro-Electric data can be used for design purposes. This is done in Figure 3 in which the \bar{N} for full suction ($\bar{N} = \bar{N} + 7$) divided by \bar{N} for no suction, Equation 5, is plotted versus the relative embedment depth, D/B . The ratio of the two \bar{N} 's is, if effect, a reduction factor which should be applied to the results of field tests on anchors embedded in soft clay. This may be done by first subtracting the quantity, $\gamma_b DA$, from the measured pullout force, dividing the result by the reduction factor of Figure 3, and then adding $\gamma_b DA$ to yield the anticipated short-term holding capacity without suction under static loading conditions. Due to inherent uncertainties involved in predicting the magnitude of the suction effect on a "shallow" anchor, when anchors are to be field tested, it is recommended that these tests be performed on deep anchors ($D/B \geq 5$) where shear strength is not a dominant parameter in determining the reduction factor.

Most seafloor clays would be considered "soft" so this suction factor

should be applied in all cases where a clay bottom is encountered. The $c = 1$ psi curve is applicable to the behavior anticipated of deep ocean clays. No research applicable to silt bottoms is available. Therefore, to be conservative, the same safety factor used with clays should also be used with silts.

Cohesionless Soils

As discussed earlier, the shear strength of cohesionless soils is purely frictional with the cohesion, c , being equal to zero. Equation 2 then reduces to

$$F_T = A \gamma_b D \bar{N}_q (0.84 + .16 B/L) \quad (6)$$

The problem in estimating the short-term holding capacity in sands is in estimating \bar{N}_q . The suggested technique for doing this is presented in Figure 4 in the form of plots of \bar{N}_q as a function of relative embedment depth, D/B , and soil friction angle, ϕ . The curved portions of the plots were derived from the theoretical work of Vesic (1969). The points at which the plots break and \bar{N}_q becomes independent of D/B are the points of separation between "shallow" and "deep" anchor behavior. In "shallow" behavior the zone of soil failure which occurs when the holding capacity is reached extends to the surface, while in "deep" behavior it does not. The points of separation between the two forms of behavior were obtained from the work of Meyerhof and Adams (1968). The independence of \bar{N}_q with respect to D/B after "deep" behavior has begun is in compliance with generally accepted concepts.

The recommended procedure for predicting the short-term holding capacity in cohesionless soil (sand) is to use Equation 6 with values of \bar{N}_q obtained from Figure 4. Techniques for estimating the soil parameters \bar{q} and γ_b will be given in a later section.

The problem of suction probably does not occur with cohesionless soil because of high permeability which allows negative gage pressures beneath the anchor to dissipate rapidly. The results of short-term field tests may be assumed to represent the proper short-term holding capacity.

LONG-TERM STATIC HOLDING CAPACITY

Cohesive Soil

The long-term static response of cohesive soils may be separated into the areas of drainage and creep.

Drainage. Drainage occurs whenever soil pore water pressures, either negative or positive gage, are set up during loading. Almost any form of loading will generate pore pressures, so water flow into or out of the soil with time is to be expected in virtually all cases. The water flow in turn will cause a change in soil density with a resulting change

in strength. Ideally a prediction of long-term anchor response would involve a prediction of the stress distribution, a laboratory analysis of the soil to determine how these stresses change the soil strength, and finally a stability analysis to determine how the holding capacity will vary with time. This type of prediction is currently impossible, and additional research is needed to determine under which conditions negative pore pressures (which lead to a decrease in holding capacity with time) are set up, the magnitude of these pressures, and the amount by which the soil strength changes with time under the influence of these pressures.

Before research of this type is accomplished, it is possible to apply a limited amount of previous work to yield approximate values for design. Several researchers (Meyerhof and Adams 1968; Kupferman, 1971) have noted the development of tension cracks on the soil surface above loaded embedment anchors. These cracks probably indicate the existence of negative normal stresses, caused by negative pore pressures, which, if allowed enough time to dissipate would lead to reduced strength in the overlying soil. The existence of these stresses is further documented by the research described in Adams and Hayes, 1967. These authors also conclude that for deeply embedded anchors positive rather than negative pressures would develop. No data is presented to substantiate this, however.

An approximate means for analyzing anchors embedded in cohesive soil under long-term drained conditions is presented in Meyerhof and Adams (1968). The technique suggested is to use the standard holding capacity equation (Equation 2) with \bar{N} and \bar{N}_g obtained using the drained strength parameters (c and ϕ) of the cohesive soil. This is done in recognition of the fact that cohesive materials behave as primarily frictional materials under long-term conditions. Two problems exist, however; one theoretical and the other practical. Theoretically there is reason to doubt that the somewhat empirical relations for obtaining \bar{N} which were developed for sands will apply to clay. The failure modes involved may differ considerably by virtue of the time element. There is reason to believe that the anchors in clay will fail progressively, thereby generating failure surfaces quite different from those produced in rapid, drained loading of sand. Even with this theoretical complication, however, the use of the standard equation with drained parameters should lead to approximately correct estimates.

The major problem is the practical one of estimating the drained strength properties of the soil. Good quality samples and careful triaxial testing would be required to yield the required parameters, and this would be expensive, time consuming, and probably not justified for most problems. Ongoing research at NCEL should provide means for approximately estimating drained properties of cohesive seafloor soils. However, for immediate use, it is necessary to resort to empirical correlations developed for soils on land in lieu of performing triaxial tests. One applicable relation is given by Bjerrum and Simons (1960) in which the drained friction angle, ϕ , is plotted versus the plasticity index.

From this plot it appears that a selection of $\bar{\phi}$ equal to 25° will be conservative for all but the most plastic soils. If the other drained strength parameter, \bar{c} , is assumed equal to 0, a conservative design is virtually guaranteed.

In summary, the following procedures are suggested for predicting the long-term holding capacities of ~~embedment~~ anchors in cohesive sea-floor soils subjected to static loads. These are ~~immediate~~ use suggestions and should be supplemented by additional future research.

1. Assume $\bar{\phi} = 25^\circ$, $\bar{c} = 0$.
2. Use Equation 6 with the estimate of $\bar{\phi}$ to calculate long-term holding capacity.
3. Calculate the short-term holding capacity using Equation 4 (with c = short-term or undrained shear strength).
4. If a short-term breakout test is performed, eliminate the suction effect according to Figure 3 to determine short-term holding capacity.
5. Compare short and long-term static capacities and use the lower value for design.

It is anticipated that the critical static situation for shallow anchors will be long-term loading and for deep anchors, short-term loading.

Creep. Many cohesive sediments are susceptible to shear creep as well as strength reduction as a result of drainage. Shear creep implies a situation whereby long-term shear straining occurs under the influence of a constant state of stress, and on land a situation known as "creep rupture" has been found to occur. In this situation the rate of shear creep increases with time until ultimately a complete failure occurs. In some soils creep rupture has occurred at stresses as low as 60 percent of the measured strength (Singh and Mitchell, 1968).

Virtually nothing is known about the creep response of seafloor soils. It is anticipated, however, that their creep characteristics will not be any worse than those of the worst terrestrial soils. Therefore, a factor of safety of 1.7 should be adequate to prevent "creep rupture" type failures. This factor of safety is probably overconservative for most installations and is recommended only for use in the design of critical or manned structures.

Cohesionless Soils

Cohesionless soils are generally not susceptible to creeping and the techniques for predicting short-term holding capacities given earlier assume that drainage occurs instantaneously. It is reasonable to assume, therefore, that long-term static holding capacity in a cohesionless soil is the same as the short-term capacity.

LONG-TERM REPEATED LOAD HOLDING CAPACITY

Embedment anchor systems which are used to moor surface vessels or buoys will be subjected to a combination of sustained and repeated loads which will vary with the tautness of the system and the nature of wave or tidal action. Experience with land soils indicates that soil-structure systems do not react in the same way to sustained-repeated load combinations as they do to strictly sustained loads of the same magnitude. In almost all cases failure occurs at a lower force level if a portion of the load is repeated. In designing anchor systems for long term use, therefore, it is necessary to consider the amount by which repeated loading will reduce the holding capacities.

There has been a good deal of research on the response of soils to repeated loading. Most of this has consisted of applying repeated loads to cylindrical soil samples in triaxial cells and determining the amount of strength reduction produced by different numbers of load repetitions. The purposes of this research were to determine how natural soils respond to earthquake loadings and how compacted soils respond to vehicular traffic. No research has been conducted to determine how natural soils respond to repeated loads extending for long periods of time. Since this may be the critical case for anchor loading, it is necessary to extrapolate the results of the existing research.

Virtually all soils respond adversely to repeated loading. However, some soils are affected more strongly than others. Lee and Fitton (1969) provide an indication of the influence that particle grain size has on the strength under repeated load conditions. Results show that soils in the fine-sand to silt range (median grain size between 0.2 and 0.02 mm) are the most susceptible to repeated loading with clays, sands, and gravels being less susceptible. Data provided in this reference cannot be used quantitatively; however, it is of value in indicating which soils are most troublesome.

Cohesive Soil

There are several reports available which provide specific information about the repeated load response of particular soil types. An extensive study of the behavior of San Francisco Bay mud, a cohesive marine soil, is described by Seed and Chan (1966). Figure 5 summarizes many of the test results obtained during this study in which pulsating stresses were applied to samples of bay mud. The plot indicates the stress state (in terms of pulsating stresses normalized by the "normal" or static undrained strength) at which failure will occur following a specified number of transient stress pulses. Values exceeding 100 can be attributed to the short duration of the pulses compared to the static strength test. As may be seen the worst situation investigated is that in which the applied stress is repeated 900 times. The resulting strength is about 60 percent of the static strength test.

Since these tests were performed to investigate earthquake response,

larger numbers of load repetitions were not investigated. This is somewhat unfortunate since it has been hypothesized (Larew and Leonards, 1962) that there is a finite, ultimate repeated load strength which applies for numbers of repeated loadings approaching infinity. It would be of interest to know how the ultimate strength relates to the 900 load repetition strength.

Data from full scale tests using screw piles subjected to repetitive loads on a soft clay land soil, Trafimenkov and Mariupolsuii (1965), indicate that strength reduction could be on the order of 50 percent, a slightly larger reduction than that indicated by the San Francisco Bay mud tests.

A laboratory study of the repeated load response of anchors embedded in clay was conducted at the University of Massachusetts (Bemben and Kupferman, 1971). The results indicated a very complicated process of upward anchor displacement with time. However, the results do not appear sufficient for quantitative design of practical anchor systems. In general a reduction factor of about 50 percent of the short term capacity appears adequate for long-term repeated loading of anchors in cohesive soil. It is suggested that this reduction factor be applied directly to other soils when additional testing is not feasible.

Cohesionless Soil

The problem of the reduction of sand strength with repeated load application is somewhat more complex. Lee and Seed (1967) investigated the response of a uniform river sand, (grain size .15 to .30 mm) placed at several different relative densities and subjected to 10 load repetitions. The specimens, tested in the undrained condition, had strength reductions ranging from 50 to 85 percent. In the same report, it is shown that 75 to 90 percent strength reductions occur after 1000 cycles.

This could be a very dangerous situation. However, one way in which the problem could become less severe would be through partial drainage. Evidence suggests that sand strength is decreased because of a buildup in pore water pressures. If these are not allowed to dissipate, the strength reduction will be extreme. In all field problems, however, at least some pore pressure dissipation will occur and therefore increase the repeated load strength. This then becomes a complex porous media flow problem which can be solved only through model and field tests.

Repeated load model anchor tests have been conducted at the University of Massachusetts (Kalajian, 1971) on a loose saturated fine to medium sand. The data are presented as the peak cyclic load normalized by the static holding capacity as a function of the cyclic creep rate. The tests were not continued long enough to establish whether cyclic creep rate dissipated; however, the data provide comparisons between "shallow" and "deep" anchor behavior under cyclic loading. The cyclic creep rate for a "shallow" anchor was considerably less than the creep rate for deeply embedded anchor, probably because of partial dissipation of pore pressures and subsequent densification of the sand in the "shallow"

case. For "shallow anchors" it was found that creep rates were negligible when the cyclic load was less than 50 percent of the static capacity. "Deep" anchors failed at lower percentages of their respective static capacities. It is reasonable to assume, however, that in no case will the holding capacity of a "deep" anchor be less than that of a "shallow" anchor simply because the anchor must be pulled through the "shallow" depth range before ultimate pullout.

Trofimenkov and Mariupolskii (1965) performed what are the only full-scale, repeated loading, pullout tests of anchors. In long-term repeated load tests on anchors embedded in fine and medium sands of loose to medium density, the holding capacities were reduced by up to 50 percent.

The test series mentioned above are the only two known to have been performed on saturated sand where drainage was allowed. Although these results are very limited, at least some tentative design procedures can be developed based on them. For "shallow" embedment, a maximum allowable cyclic load of 50 percent of the static or short-term capacity is recommended. For "deep" embedment a conservative design should result if the applied cyclic load is less than 50 percent of the static capacity corresponding to the transition between "deep" and "shallow" behavior.

It is possible that the required reduction factor may be greater with soil in the silt-fine sand range. It is suggested that seafloor soil grain size characteristics be determined whenever a direct embedment anchor is to be established in granular soil which will be subjected to repeated loadings. An anchor in granular soil with a characteristic mean grain size D_{50} , greater than 0.20 mm should be designed with the repetitive loading factor given above. If the soil falls in the silt-fine sand range, D_{50} between .02 mm - .20 mm, it may be necessary to use a different anchoring technique or employ high safety factors (a minimum of 10). Another possibility would be to reduce system tautness and thereby reduce the effect of surface wave action and dampen repetitive loading. The approach depends upon system requirements, system importance, and the consequences which would result from a failure. Work is on-going at the University of Massachusetts under a NCEL contract to evaluate the long-term repeated load response of anchors embedded in soil in the silt-fine sand range.

ESTIMATION OF SOIL PROPERTIES

In order to use the prediction equations which have been given, it is necessary to have estimates of several soil properties. Aside from the drained strength parameters, c and ϕ , which were discussed previously, the pertinent soil parameters are the soil buoyant unit weight, γ_b , the undrained shear strength (for a cohesive soil), c , and the angle of internal friction (for a cohesionless soil). There are two major problems involved in estimating these quantities. First it is necessary to estimate the distribution of these properties at the proposed anchor

site; and second, it is necessary to select a characteristic shear strength, density, or friction angle (for use in the equations) given a possibility of strong variation of these quantities with sediment depth.

Considering the first problem, it would be preferable either to obtain a good quality core and perform laboratory tests, perform a meaningful in-situ test, or undertake a combination of in-situ and laboratory testing. If this sort of program cannot be accomplished, it is recommended that at least the type of bottom (cohesive or cohesionless) be determined, either by observing a disturbed grab sample or through a careful geologic interpretation of the general area.

If the general sediment type is determined to be cohesive, then the use of the soil properties (c and γ_b) illustrated in Figure 6 is recommended. These properties are low for normally consolidated deep water clays and, when used with Equations 4 and 5, should provide conservative estimates of holding capacities. An exception to the use of Figure 6 would be a region of rapid sediment deposition, such as an active river delta. In this case the soil properties should be measured directly because extremely soft, underconsolidated clays may be encountered.

If the bottom is determined to be a cohesionless soil, then the use of an angle of internal friction, ϕ , of 30° and a buoyant unit weight, γ_b , of 60 pcf is recommended. It may be necessary to use these conservative values in almost all sandy bottom situations since good quality sampling of sand is very difficult.

Given these property distributions it is still necessary to approach the second problem of selecting characteristic or average values for use in the holding capacity equations. This problem is greatest with cohesive soils since their property variations are generally larger. Experience with these soils indicates that reliable results may be obtained if the strength and density are averaged over the entire depth of embedment for a "shallow" anchor. For a "deep" anchor however, there is conflicting data on the depth range over which the strength should be averaged. The data do indicate, however, that conservative predictions should result if the strength is averaged over a zone above the anchor fluke with a thickness that is the same as the depth at which "deep" behavior begins.

For a uniform strength profile, of course, the appropriate characteristic strength, c , to use in holding capacity calculations is the measured strength. For a profile in which the strength increases linearly from near zero at the surface, the statements of the preceding paragraph lead directly to the curves of Figure 7. The quantity D/B is determined along with the strength, c , at a depth, D , (i.e., at the anchor fluke). Figure 7 is entered and the parameter D/B is obtained. The characteristic strength, c , for use in predicting holding capacity is taken as the strength at a distance D above the anchor fluke. For more complex profiles a trial and error procedure may be required to determine the characteristic strength. To simplify the problem, profiles

should be reduced to either uniform or linearly increasing strength whenever possible.

The same sort of characteristic property selection technique would probably also be valid for sand. However, since the determination of the variation of sand properties is difficult, this procedure is not recommended. Rather, the standard properties listed previously should be used.

SUGGESTED PREDICTION PROCEDURE

In order to simplify the holding capacity prediction process, the suggested procedures which were discussed previously are listed in this section without rationale. This general procedural framework is shown by the block diagram of Figure 8 with each item of the diagram being discussed briefly below. The numbering system below compares with that of the diagram.

In virtually all cases, an anchor should be installed so as to display "deep" behavior. In all of the curves of holding capacity or holding capacity parameters versus depth, there are breaks below which the holding capacity increases less rapidly. This behavior in the lower sections of these plots is termed "deep", and it is advantageous to establish a "deep" anchor because errors in locating the anchor, either during installation or because of deformations after installation, do not cause large changes in holding capacity. The anchor is, therefore, more reliable.

Hansen (1953) has shown that holding capacity may increase up to 25 percent in clay if plates are rough rather than smooth. Therefore, to improve holding capacity plates could be ritted, grooved, or simply allowed to rust.

A step by step approach for calculating anchor holding capacity is as follows:

1. Determine Design Parameters. Determine the anchor fluke embedment depth, D , width, B , length, L , and area, A . In a typical design problem, the anchor dimensions would be trial values. The engineer would proceed through the calculations to obtain an estimated holding capacity and then determine if it satisfies the design criteria for the anchored system. If not, an iterative procedure would follow with different anchor system parameters being tried until a satisfactory solution was developed.

Estimating the embedment depth, D , may become a major problem in itself depending upon the means used for anchor installation. Typical pile driving equations may be used approximately for vibratory and impact installation; and the techniques of NCEL (1971) may be applied to free fall and, very approximately, to explosive embedment. Research currently underway at NCEL will provide improved techniques for predicting penetration behavior.

2. Determine Soil Type. Determine the general soil type (cohesive or cohesionless). This will be obvious from the visual observation of a bottom sample, even a very disturbed grab-type sample. In areas far from shore, it may be possible to estimate the bottom type given a chart of the regional geology. In addition, good geophysical data, if available, may give clues. If at all possible, however, a bottom sample should be obtained.

3. Determine Short-Term Holding Capacity for Cohesive Soil. Steps 3 through 5 assume the soil has been determined to be cohesive. The procedure to be followed in estimating the short-term holding capacity depends upon whether or not good quality cores or in-situ strength data have been obtained or a field test has been performed.

Core or in-situ data available. If reliable engineering properties are available, the procedure is as follows:

(a) Plots of the undrained or vane shear strength and unit weight distributions should be developed. If the strength and density are approximately uniform with depth, then the characteristic strength, c , and characteristic density, γ_b , are simply the mean values over the depth range, D . If the strength increases approximately linearly with depth from a value of near zero at the seafloor surface, then the plots of Figure 7 should be used to obtain the characteristic strength and density. This is done by first calculating D/B and taking the strength, c , at depth, D , from the strength profile. Figure 7 is entered with these values and the quantity D/B is determined. The characteristic strength, c , and density are then taken as the strength and density a distance, D , above the anchor fluke. For more unusual strength and density profiles, either a conservative uniform or linearly increasing curve should be drawn through the data or an experienced seafloor soils engineer should be consulted.

(b) Given D/B and c , the parameter \bar{N}_c is obtained either from Figure 2 or Equation 5.

(c) The short-term holding capacity is calculated from Equation 4.

Soil data unavailable. If strength and density profiles are not measured, then the profiles of Figure 6 should be used and Steps 3a through 3c repeated. This procedure may be simplified by using Figure 9 and obtaining holding capacity, F_T , directly. It should be noted that in almost all cases, this procedure will yield unnecessarily conservative holding capacities. If at all possible, strength and densities for the design location should be measured.

Field test. In some research and practical situations, it may be

necessary to use the results of short-term field tests to obtain design holding capacities. This may be a good means of reducing uncertainties; however, it is necessary to modify the measured capacities to account for suction forces, or unconservative design values will result.

Figure 3 may be used to account for the suction effect. Using D/B and an estimate of c (1 psi should be a reasonable value in most cases), a reduction factor, R , is obtained. This is inserted into the equation given on the figure and the design short-term holding capacity, F_T , is calculated. An estimate of the soil unit weight, γ_b , is needed and may be assumed equal to 25 pcf in most cases.

4. Determine Type of Loading. Most anchor trial tests, salvage work, and other projects which require a reaction force for a short period of time are considered to be short term static loadings. Surface vessels and buoys generally exert a long-term repeated loading condition, although certain designs may convert the repeated load into a virtual long term static condition. Subsurface buoys, suspended arrays, and other suspended structures exert long term static loads.

(a) If the loading is short term static, the design holding capacity is F_T as determined in Step 3c or 3d.

(b) If the loading is long term repeated, the design holding capacity is one-half F_T from 3c or 3d. This capacity refers to the characteristic peak repeated load.

5. Determine Long-Term Holding Capacity. If the loading is long term static, the long term or drained capacity must be estimated.

(a) The drained friction angle, $\bar{\phi}$, may be obtained from a triaxial or similar shear test on a high quality sample. If such a test is not performed then a conservative value of 25° may be assumed for most cohesive soils.

(b) Figure 4 is entered with $\bar{\phi}$ and D/B , and the parameter \bar{N}_q is obtained.

(c) The drained holding capacity F_{TD} is obtained from Equation 6 (substituting F_{TD} for F_T).

(d) F_{TD} from Step 5c is compared with F_T from Step 3c or 3d and the lower value is used as a design holding capacity. If the anchored system is critical or manned the result should be multiplied by 0.6 to account for possible creep effects.

6. Determine Short-Term Holding Capacity for Cohesionless Soil. Steps 6 through 9 assume the soil has been determined to be cohesionless. The procedure to be followed in estimating the short-term holding capacity depends upon whether good quality cores or in-situ strength data have been obtained or a field test has been performed.

Core or in-situ data available. If this information is available,

the procedure is as follows:

- (a) The friction angle, ϕ , and unit weight, γ_b , in the vicinity of the anchor fluke should be estimated (this may be difficult).
- (b) The parameter N is obtained from Figure 4, given ϕ and D/B .
- (c) The short-term holding capacity, F_T , is obtained from Equation 6.

Soil data unavailable. If strength data are unavailable, then the use of a friction angle of 30° and a unit weight of 60 pcf is recommended. Design curves using these parameters are presented in Figure 10. The short-term static holding capacity, F_T , may be taken directly from the curves given L , B , and D . Linear interpolation between the curves may be used for values of B not presented.

Field test. The measured holding capacity from a field test can be considered to represent the proper short-term holding capacity, because suction will not be significant in cohesionless soil.

7. Determine Type of Loading. The type of loading should be determined in a manner identical to that of Step 4 for cohesive soils. If the loading is short or long-term static, the estimated design holding capacity is F_T as calculated in Step 6c, 6d, or as measured in a field test.

8. Determine Grain Size. If the loading type is long term repeated, a grain size analysis of a disturbed sample should be performed. If the median grain size is found to lie between .02 and .2 mm, either a different mooring system design should be developed (i.e., one which reduces effects of repeated loading) or high factors of safety (greater than 10) should be used.

9. Determine Anchor Relative Embedment Depth. For other grain sizes, it is necessary to determine whether the anchor is to be considered "deep" or "shallow". This may be done by referring to either Figures 4 or 10, and determining whether the particular range of design parameters places D/B below or above the sharp breaks in the curves.

(a) If the anchor is "shallow" the design repeated load holding capacity is one-half F_T from 6c or 6d.

(b) If the anchor is "deep", it is necessary to calculate the short-term holding at the D/B at which "shallow" behavior changes to "deep". This D/B corresponds to the break points in the curves of Figures 4 and 10. The same values of B , L , ϕ , and γ_b as used previously should be used and Step 6 should be repeated with the new D/B . One-half of the short-term holding capacity calculated using these parameters should be used for design purposes.

SAMPLE PROBLEM

An example of an application of the suggested prediction procedure follows. The step identifications are identical to those of the preceding section.

Problem

A direct embedment anchor with a 3-foot-wide square fluke has been placed to an embedment depth of 15 feet in a cohesive soil deposit. A good quality core has been obtained and the measured vane shear strength profile may be approximated by the curve of Figure 11. The buoyant unit weight has been measured and found to be about 35 pcf throughout the profile.

The anchor is to provide support for one leg of a suspended sub-surface array which is to be in service for several years. Determine the design holding capacity of the anchor.

Solution

1. $D = 15$ feet
 $B = 3$ feet
 $L = 3$ feet
 $A = 3 \times 3 = 9$ square feet
2. The general soil type is cohesive.
3. A core is available.

(a) The vane strength distribution is as shown in Figure 11. To determine the characteristic strength it is necessary to use Figure 7. D/B is calculated to be $15/3 = 5$. The strength at the anchor, c_a , is found from Figure 11 to be 3 psi. From Figure 7 (using linear interpolation between the $c_a = 2$ and $c_a = 3.5$ lines), the quantity D/B is found to be 1.75. Multiplying by c_a , D_c is determined as 5.25 feet. This is the distance above the anchor at which the characteristic strength, c , is to be found. Referring to Figure 11 again (and a depth of $15 - 5.25 = 9.75$ feet), c is found to be 2.0 psi.

The buoyant weight, γ_b , is 35 pcf.

(b) Using Equation 5:

$$N_c = 3.8 (D/B) \left(\frac{0.7}{c} + 0.3 \right)$$

$$N_c = 3.8 (5) \left(\frac{0.7}{2.0} + 0.3 \right)$$

$$N_c = 12.38$$

or 9 whichever is smaller.

Since 9 is smaller

$$N_c = 9.0$$

This represents a "deep" anchor.

(c) Using Equation 4:

$$F_T = A (c\bar{N}_c + \gamma_b D) (0.84 + .16 B/L)$$

$$F_T = 9 [2(9)(144) + 35(15)] [0.84 + .16 (\frac{3}{8})]$$

$$F_T = 28,000 \text{ pounds}$$

This is the estimated short-term holding capacity.

4. The load is to be applied for several years and apparently the loading will be relatively constant. Therefore, this is a case of long-term static loading.

5. (a) No triaxial tests were performed. Therefore the drained friction angle ϕ is estimated conservatively to be 25° .

(b) Using Figure 4 with $\phi = 25^\circ$ and $D/B = 5$, \bar{N}_q is found to be 4.5

(c) Using Equation 6:

$$F_{TD} = A \gamma_b \bar{DN}_q (0.84 + 0.16 B/L)$$

$$F_{TD} = 9 [35(15) 4.5] [0.84 + 0.16 (\frac{3}{8})]$$

$$F_{TD} = 21,262 \text{ pounds}$$

This is the estimated long-term static holding capacity.

(d) The long-term holding capacity is found to be less than the short-term capacity. Therefore the long-term case is critical and should be used in design. The estimated holding capacity for design purposes is then about 21,000 pounds. If the structure were especially critical or manned, this quantity would be multiplied by 0.6 to account for possible creep effects.

SUMMARY AND CONCLUSIONS

1. The state-of-the-art of predicting direct embedment anchor holding capacity is inexact at present. This report was written to maximize the usefulness of existing related research so that immediate use predictions can be made. The predictions given are not intended to be final but rather a "best guess" given the present state of technology.

2. It is important to subdivide the problem of predicting embedment anchor holding capacities into several areas because the same anchor may produce significantly different capacities under different loading and soil conditions. Two general soil types, cohesive and cohesionless, and three loading conditions, short-term, long-term static, and long-term repeated, were selected for investigation. Dynamic loading and

rock were not considered.

3. Considerable research on short-term capacity in cohesive and cohesionless soils has been accomplished. Procedures for predicting this capacity based on empirical results appear to be relatively far advanced and accurate.

4. Holding capacities under long-term repeated and long-term static loading conditions are not well understood at present. It is necessary to combine work from other areas with a small amount of directly applicable research to yield approximate, immediate use results.

5. Given these limitations, the procedures of this report are recommended for use at the present time. Additional research is strongly recommended.

RECOMMENDATIONS FOR FURTHER RESEARCH

1. The holding capacity of embedment anchors under static and repeated long-term loading is poorly understood at present. Of these two, static long-term is the less complex and, therefore, should be investigated first. Since cohesive soils are more common and also more susceptible to long-term effects, they should be investigated in greatest detail.

2. The central element in researching long-term static capacities in cohesive soils should be the small to medium scale laboratory model test. It is relatively economical, can be well controlled, and a large number of parameters can be measured. The following types of experiments are recommended:

(a) Several cohesive soils, each at several densities, should be investigated.

(b) Several fluke sizes and embedment depths should be used.

(c) The soils themselves should be tested extensively to determine undrained strength, drained strength, and creep strength.

(d) Short-term holding capacities should be determined first; and then loads a given percentage of the short-term loads should be applied and the time to breakout measured.

(e) Upward anchor movement and pore pressures at several points in the soil should be measured.

3. Analytical research, possibly using finite element computer programming, should be performed so that improved theoretical predictions of stress distributions and failure loads can be made. These should be compared with the results of the model tests and modified if necessary to comply with the empirical data.

4. When an adequate prediction technique has been developed, it should be tested with a limited number of well monitored, long-term static load, full scale, field tests.

5. When long-term static holding capacity is sufficiently well

understood, a program, similar to the above, for investigating long-term repeated loading should be executed.

6. The problem of penetration prediction also deserves additional research along similar lines. Much of this is already underway at NCEL.

7. The reduction in holding capacity during earthquakes could be severe for anchors embedded in silts; research is needed to calculate the magnitude of the problem.

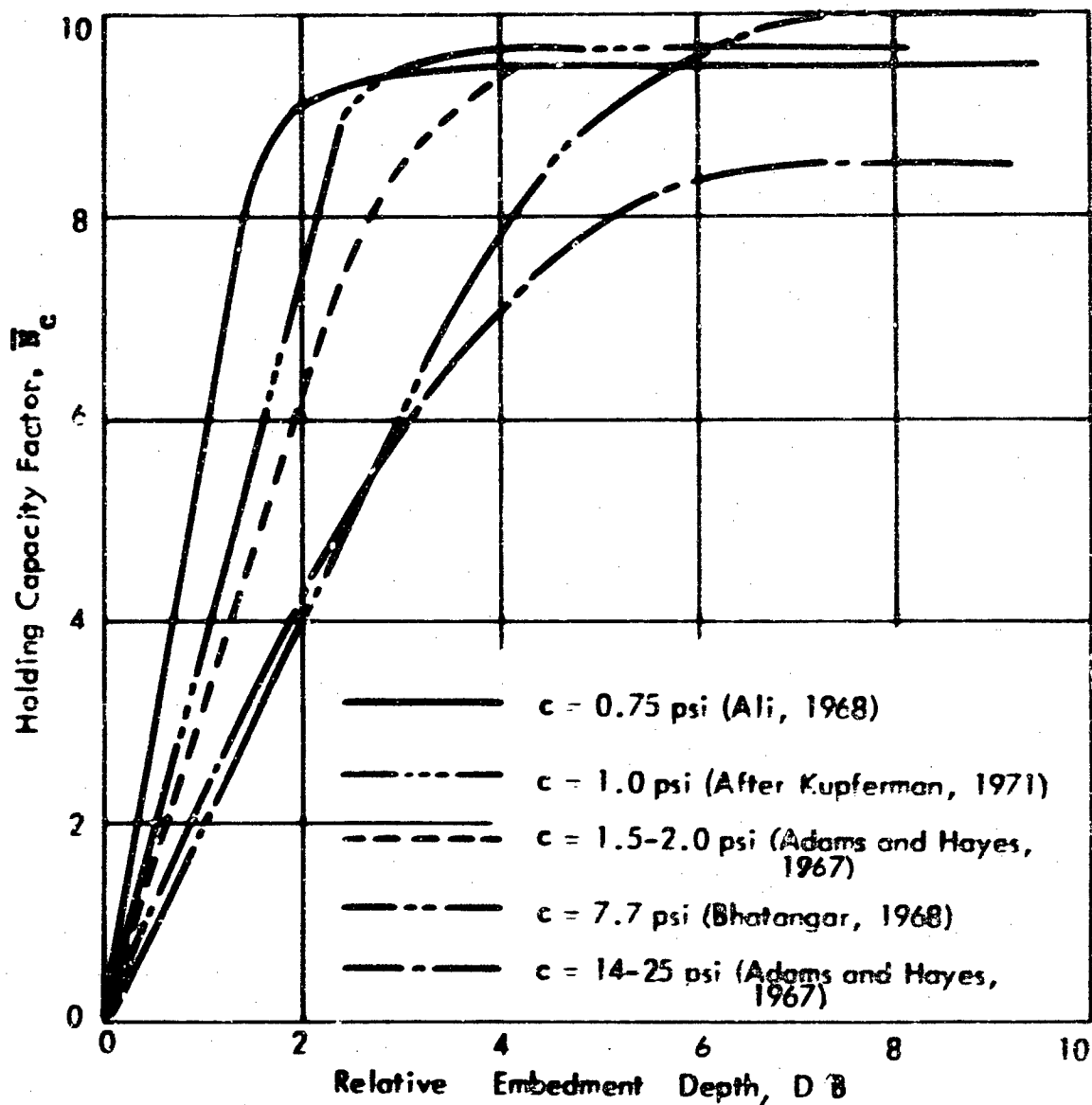


Figure 1. Holding capacity factor \bar{N}_c versus relative embedment depth D/B as measured in laboratory tests.

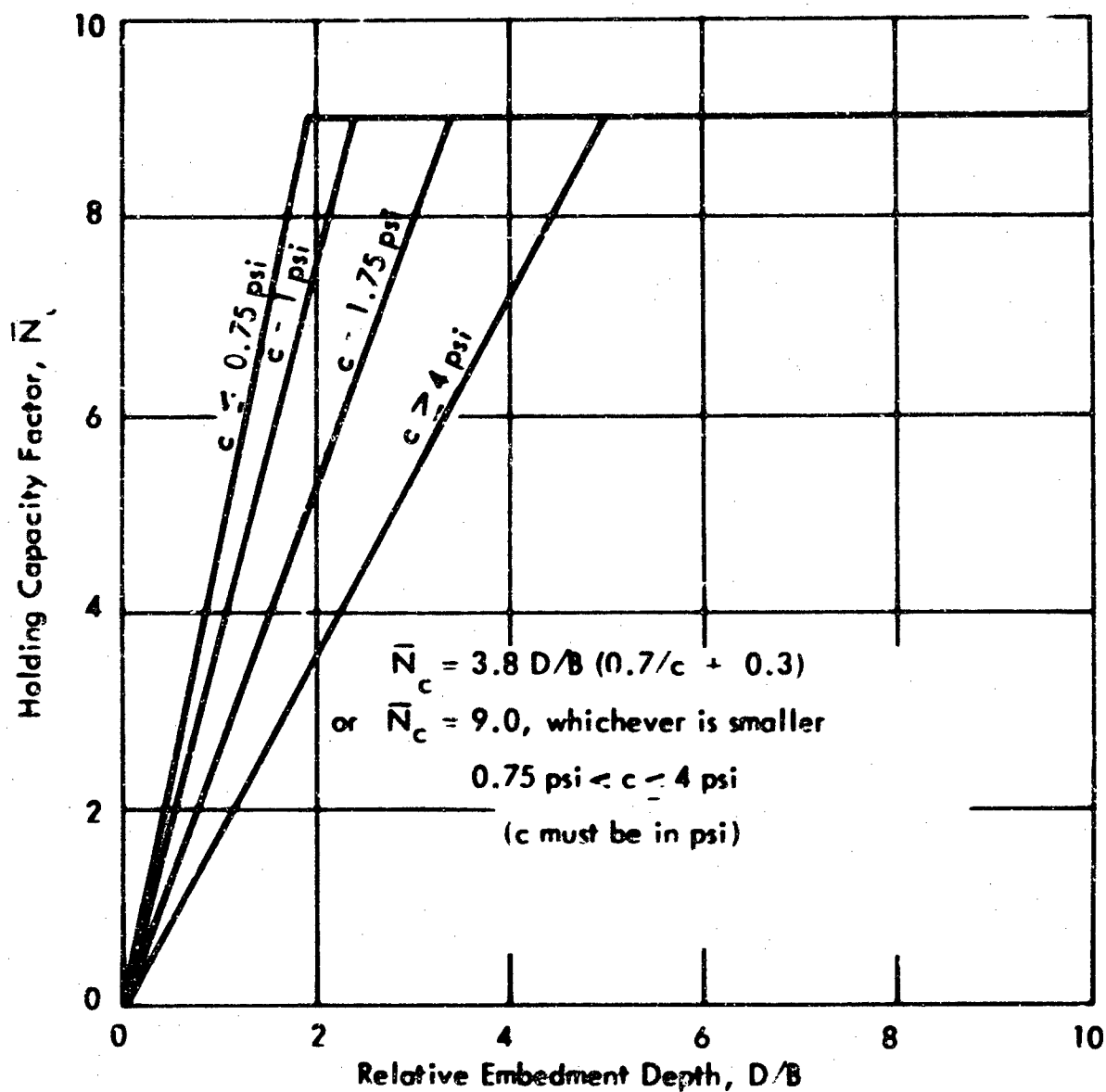


Figure 2. Design curves of holding capacity factor, \bar{N}_c , versus relative embedment depth (D/B) approximated from Figure 1.

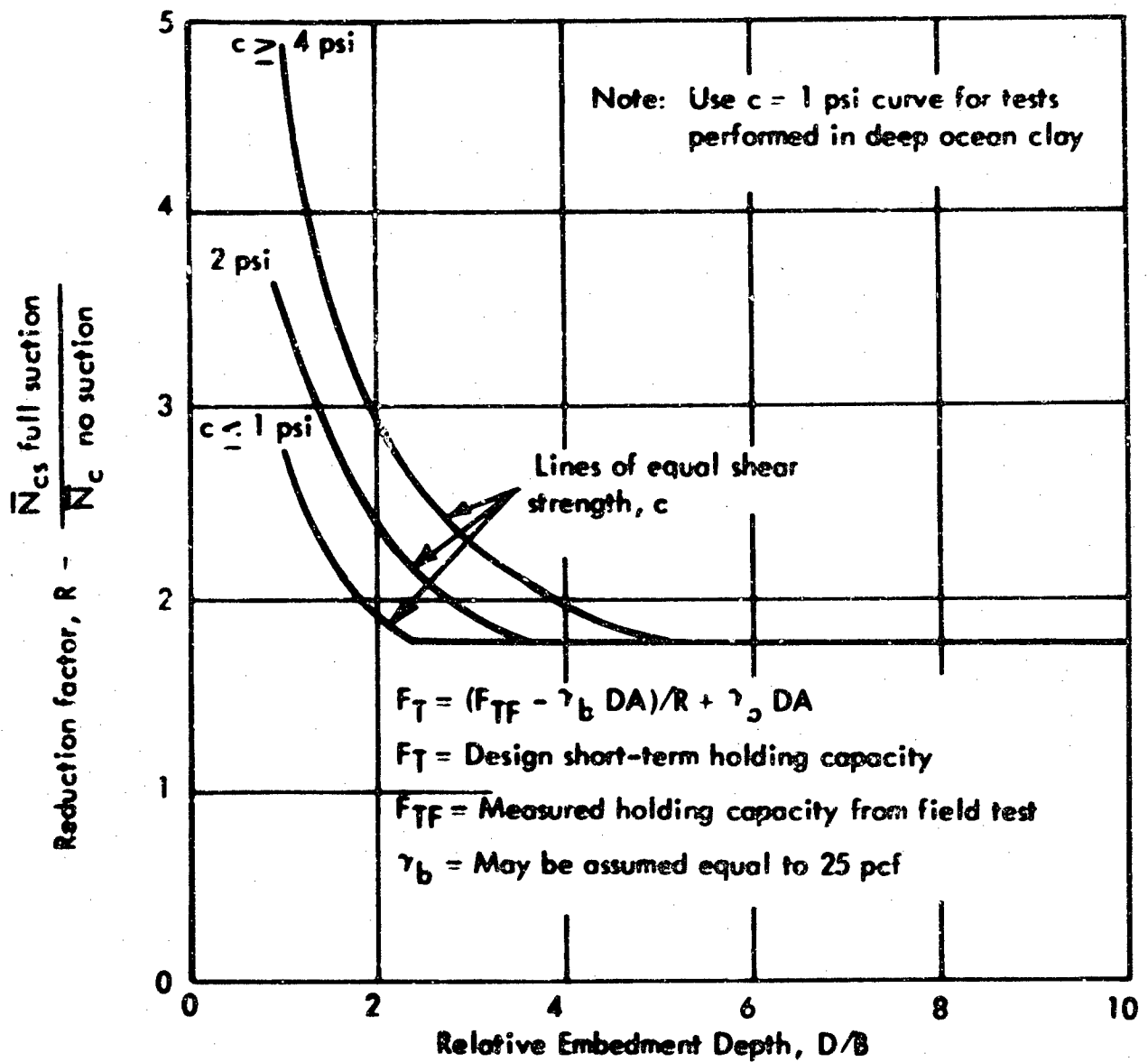


Figure 3. Reduction factor to be applied to Field Anchor Tests in clay soils to account for suction effects.

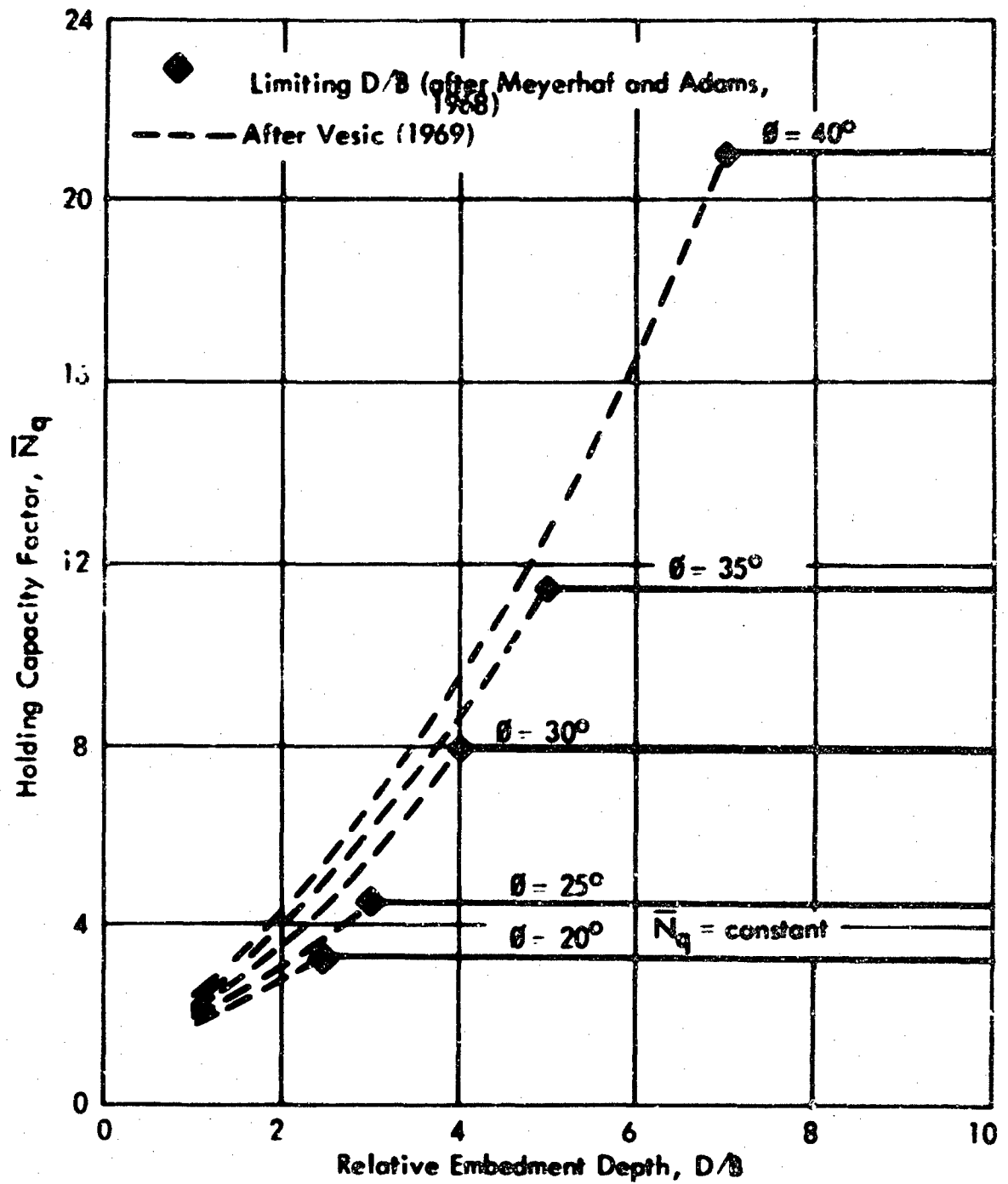


Figure 4. Holding capacity factor, \bar{N}_q , versus relative depth for cohesionless soil, $c = 0$.

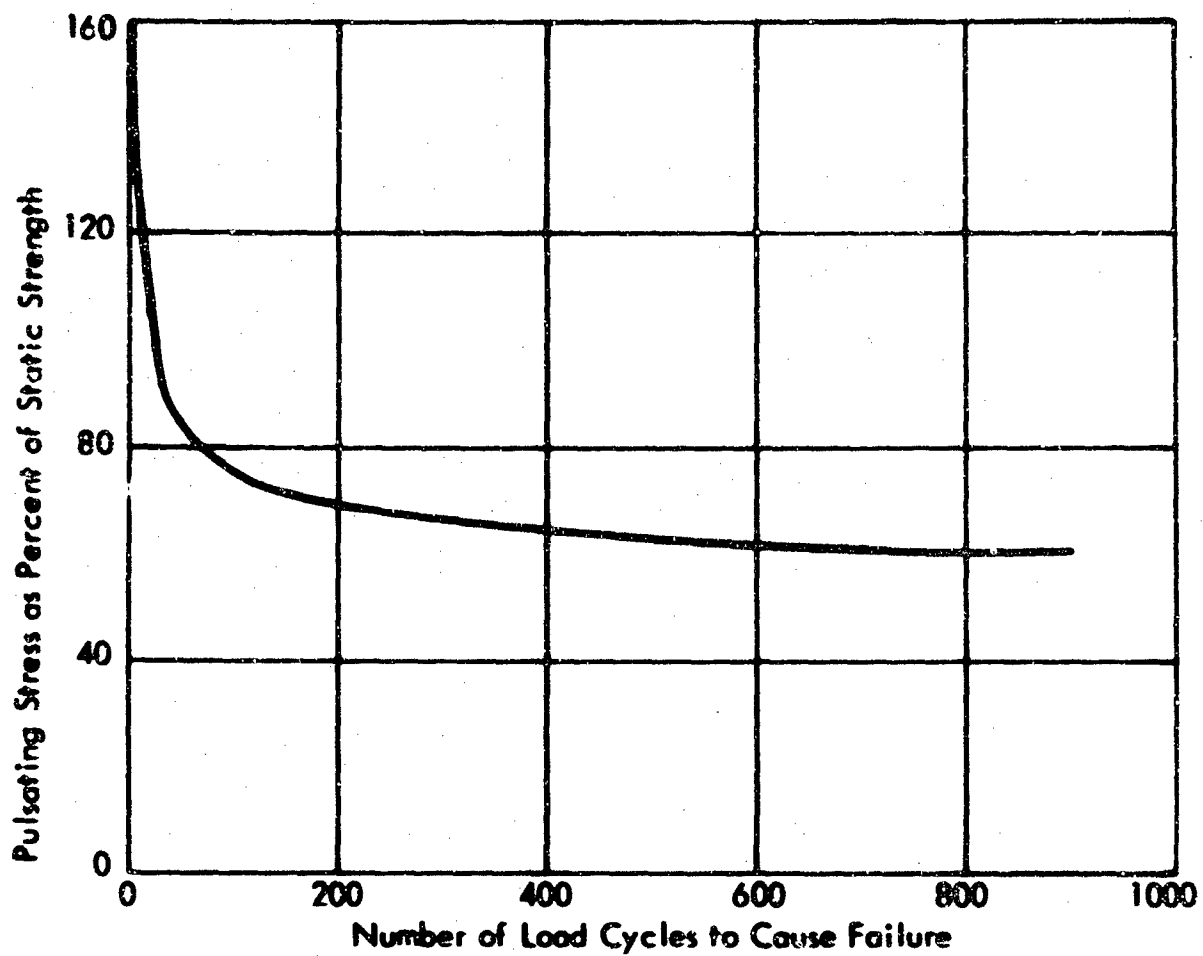


Figure 5. Relationship between number of load cycles and pulsating stress as percent of normal stress for San Francisco Bay mud. (after, Seed & Chan, 1966)

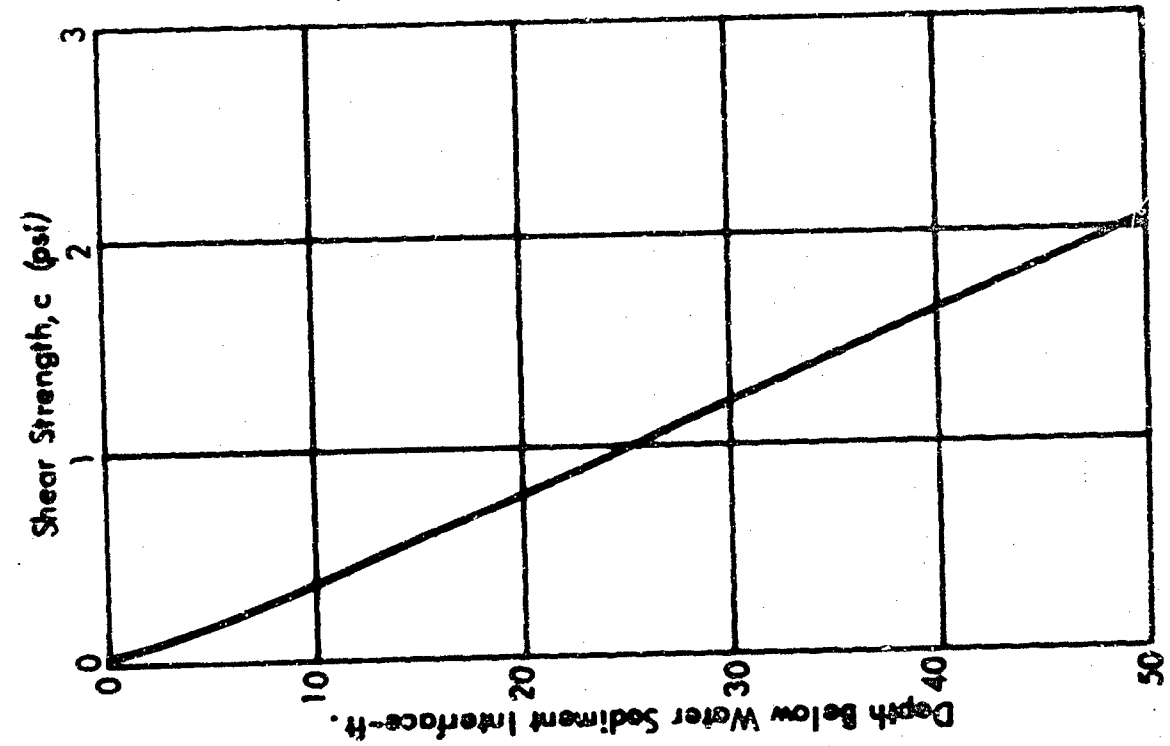
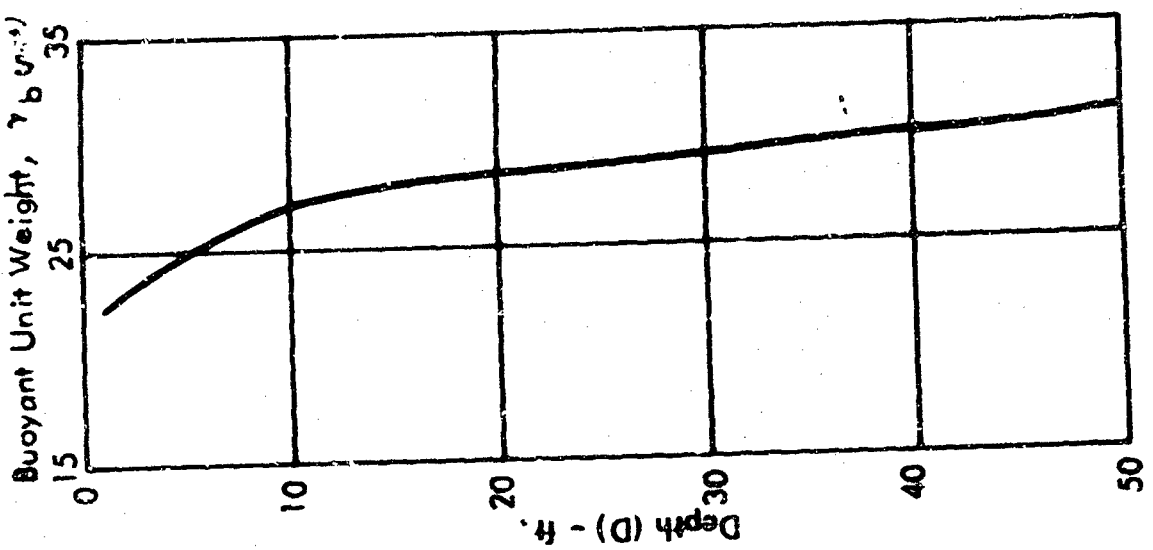


Figure 6. Recommended properties of a deep-water clay.

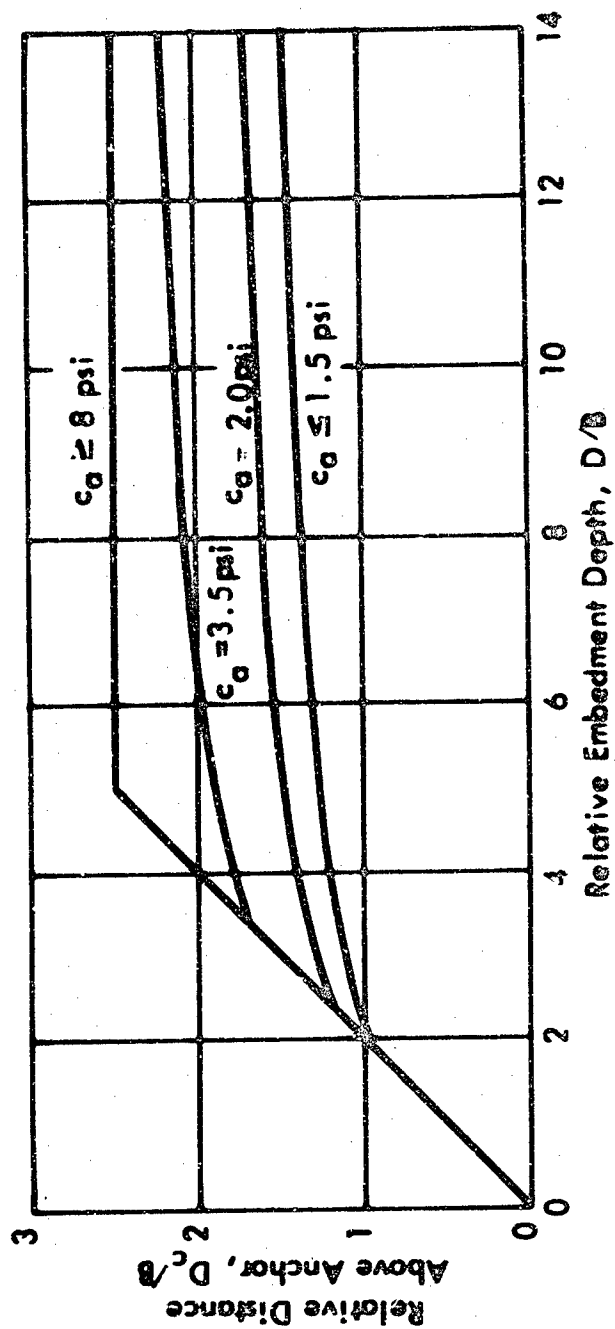


Figure 7. Plot for obtaining, D_c , the distance above the anchor at which the characteristic strength, c_g , is to be taken.

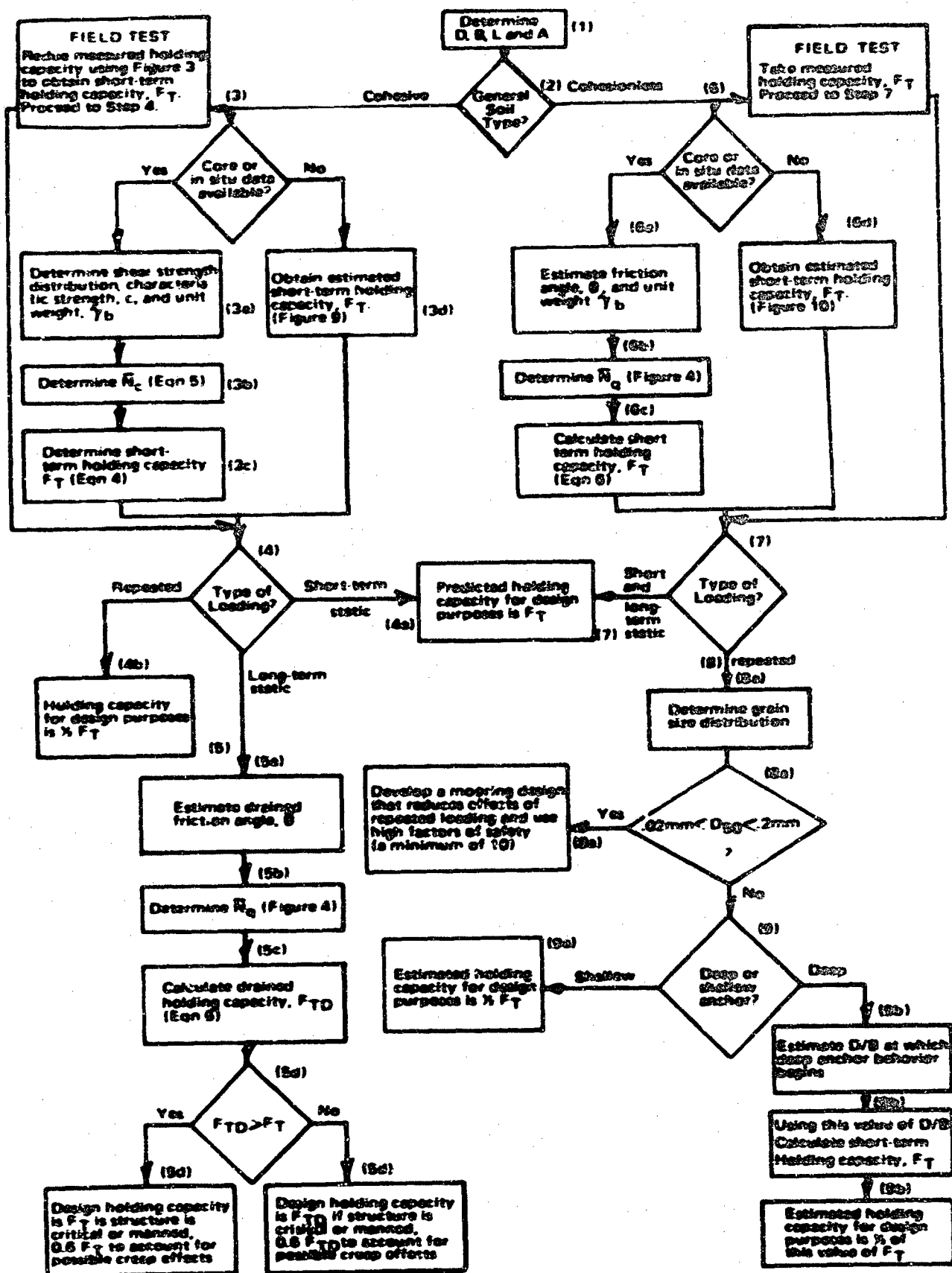


Figure 8. Block diagram illustrating suggested procedure for predicting embedment anchor holding capacity.

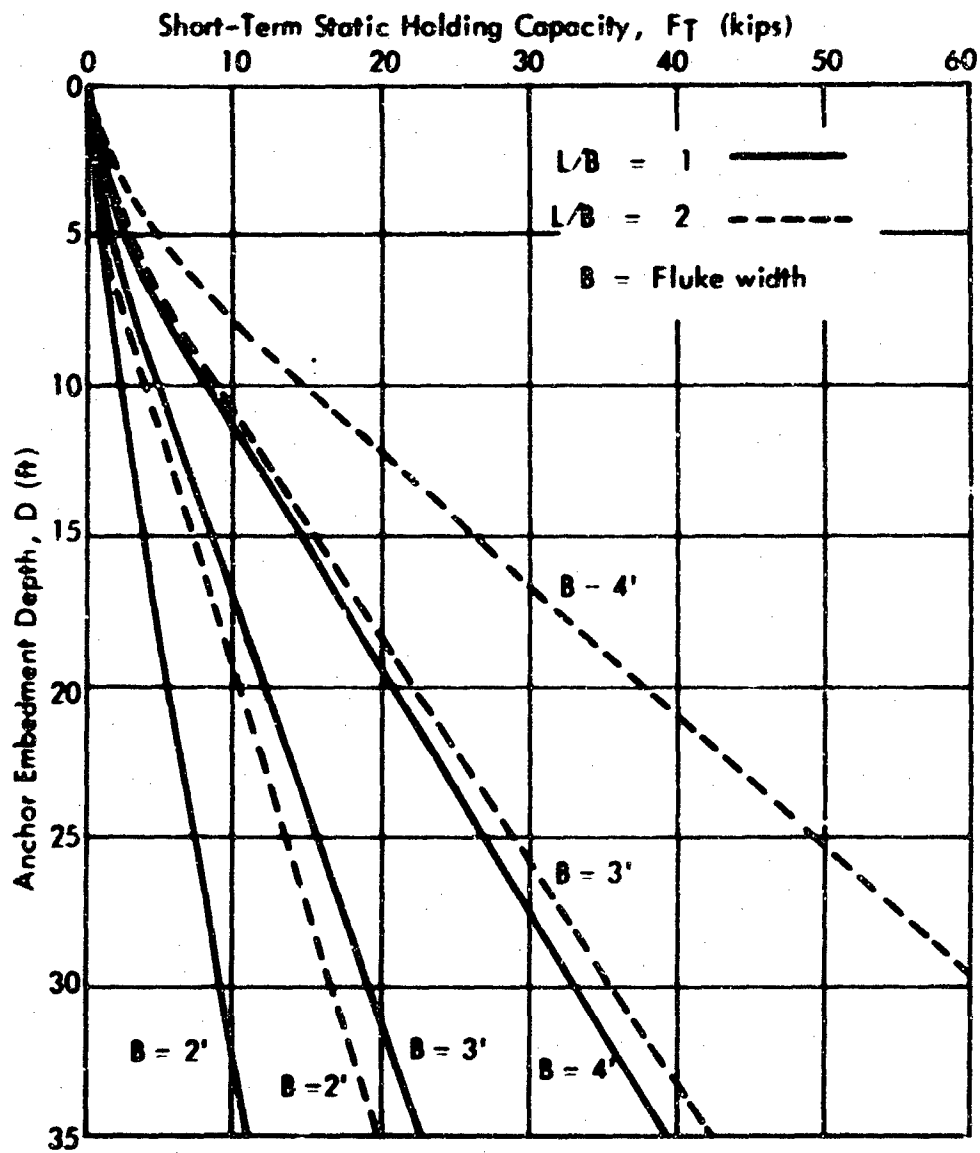


Figure 9. Short-term holding capacity versus depth for a cohesive soil with properties given by Figure 6.

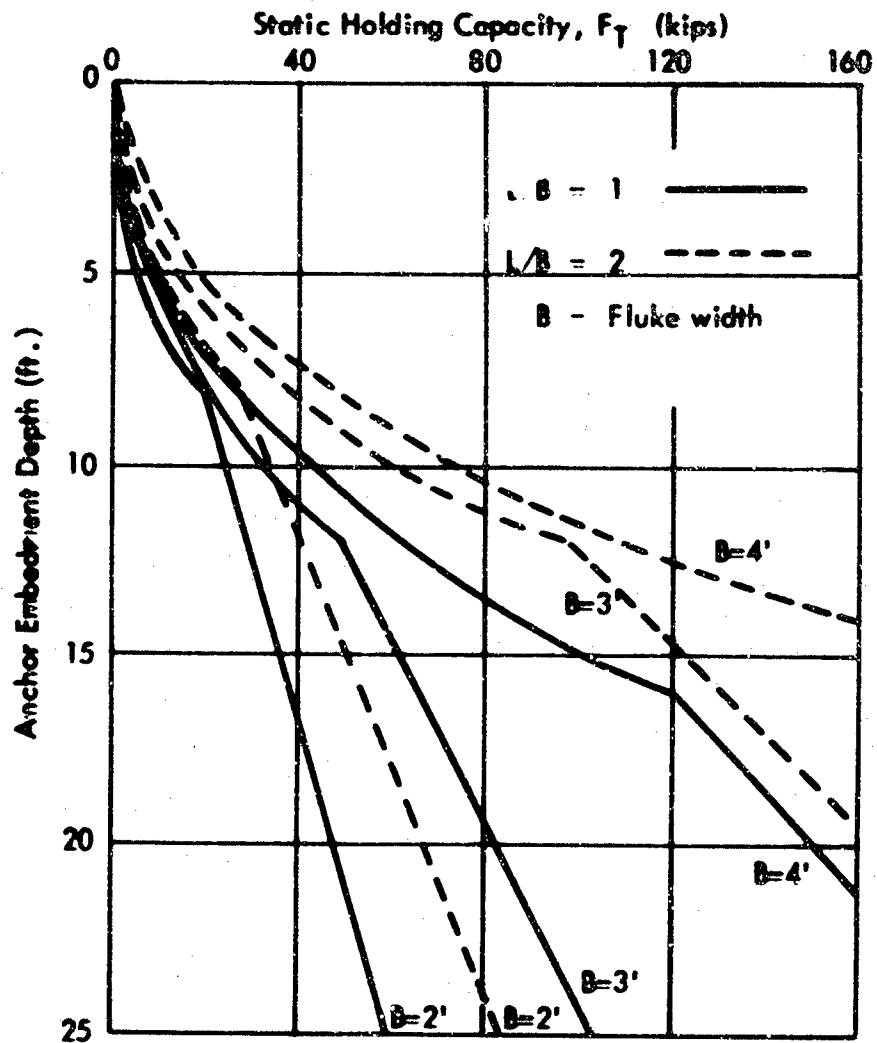


Figure 10. Holding capacity versus depth for a cohesionless soil, $c = 0$, $\phi = 30^\circ$, $\gamma_b = 60$ pcf, for anchor plates $L/B = 1$ and 2.

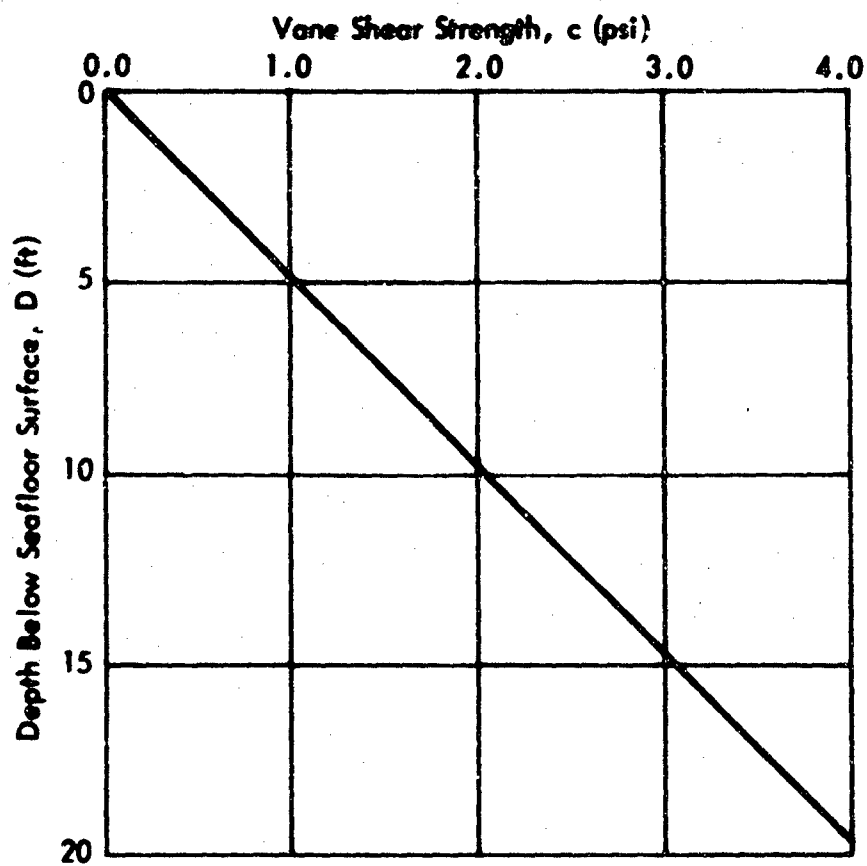


Figure 11. Vane strength profile for sample problems.

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NOMENCLATURE

A	Fluke area (sq ft)
B	Fluke width or diameter (ft)
c	Cohesion (undrained shear strength) of a cohesive soil (psi)
\bar{c}	Cohesion intercept under drained shear conditions (psi)
c_a	Cohesion (undrained shear strength) at anchor fluke (psi)
D	Embedment depth of fluke (ft)
D_c	Distance above anchor at which characteristic strength (cohesion) is measured (ft)
D_{50}	Soil median grain size (mm)
F_T	Anchor holding capacity under short-term static conditions (lb)
F_{TD}	Anchor holding capacity under long-term static conditions (lb)
L	Fluke length (ft)
N	Normal stress on failure surface (psi)
\bar{N}_c, N_q	Holding capacity factors
\bar{N}_{cs}	Holding capacity factor \bar{N}_c assuming full suction
γ_b	Soil buoyant unit weight (pcf)
τ_f	Shear stress at failure (psi)
ϕ	Angle of internal friction (degrees)
$\bar{\phi}$	Angle of internal friction for cohesive soils during drained (long-term) shear (degrees)